Flow, Suspended Sediment Transport, and Bank Deposition in the Colorado River During the 1996 Test Flood

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Abstract

Flow and suspended sediment transport data from the late spring 1996 test flood are used to evaluate the response of the riverbed and river edge sand deposits along the Colorado River through the Grand Canyon to this high flow event. Reach-averaging was found to produce a useful framework for this purpose. The falling limb of the test flow hydrograph had previously been used to improve a reach-averaged flow routing model. Further improvements using both the falling and rising limbs are made, then hydrographs from the improved flow model are compared to ones computed from stage measurements made at a series of temporary pressure gages. The discrepancy between these hydrographs is used to calculate the rate of sand deposition as a function of time. This calculation demonstrates that most of the sand was deposited during the first 13hrs of the test flow. The sand volume represented by the area under the reach-averaged, rate-of-sand-deposition curve (18 m^3/m) is shown to nearly fill the open depositional sites (25 m^3/m), justifying the use of a simple reach-averaged sand deposition model. This model also can be used to calculate the long-term losses of sand along the edges of the channel and indicates that about 30% of the sand deposited by the test flood was returned to the riverbed in the following several months. The suspended sediment data from three gage stations is used to show that the amount of sand on the bed of the river had come to equilibrium with the flow over at least a 169 km reach of the river (Little Colorado River to National Canyon) after the first day of the event, and that the area of the channel bed covered by sand decreased approximately exponentially in time everywhere within this 169 km segment. Finally, the amount of sand deposited on the channel edges is compared to the amount of sand exported from the river to Lake Mead. During

the first half day of the test flow approximately 96 times as much sand was deposited on the channel edges between Lees Ferry and Diamond Creek as was transported into Lake Mead; however, over the entire test flow this ratio dropped to 22 because after the first day the depositional sites along the river were essentially full but the loss to Lake Mead continued, although at a decreasing rate. Both of these factors would increase by more than 25% if the sand deposition between Diamond Creek and Lake Mead were included.

Key phrases: modeling flow; modeling suspended sand transport; deposition rates

Key words: erosion; deposition ; Colorado river; Grand Canyon; Glen Canyon Dam

Introduction

The Glen Canyon Dam has reduced both the peak flow of the Colorado River through the Grand Canyon and the amount of fine sediment (sand, silt, and clay) conveyed by the Colorado to Lake Mead. This has resulted in a new equilibrium with respect to the riparian and riverine sand deposits along the channel. As a result, sand has been lost from the riparian zone everywhere below the Dam, but the loss has been especially severe in Marble Canyon. (See Patten et. al., this volume, for locations along the river.) Several geomorphologists, including the authors, have argued that some of this sand could be replaced by a flow of $1200 m^3/s$ lasting for the better part of a week or a flow of $1400 m^3/s$ lasting for a few days. A test flow was carried out in the early spring of 1996 to verify this and other hypotheses (Patten et. al., this volume).

The design of the test flow was conservative in the sense that sufficient high-flow duration was provided so that a lower than estimated deposition rate would still result in a deposit that was large enough to be measured accurately. Nevertheless, our pre-flood hypothesis with respect to this issue was that it would take several days and perhaps a good fraction of a week before the new topography would come to equilibrium with the flow. Similarly, based on previously constructed models that embodied data from the research flows of 1991 we hypothesized (1) that sand would be carried from the bottom to the sides of the river as long as there were open depositional sites along the river margins, (2) that eventually all of the open depositional sites along the river would fill, and (3) that the fraction of the riverbed covered by sand would decrease monotonically as sand was transported to the edges of the river and was carried downstream to Lake Mead. In an operational rather than an experimental mode, the management procedure would have been to terminate the high flow after most of the deposition on the river margins had occurred, but before much sand had been carried out of the system. In order to design a flow that would do this optimally, the rates of deposition on the river margins had to be determined.

In this paper, our first goal for the test flood was to confirm the above-stated hypotheses concerning the nature of the flow and suspended sediment transporting system. The second goal was to determine the rates at which the system adjusted to the new (high flow) equilibrium. A third goal was to obtain some constraints on the reach-averaged loss of newly deposited sand after the test flow, although the main task of determining the local and overall rates of sand deposit loss fell to the group responsible for beach and channel surveying. The methods used in this paper are based on the premise that measurements of river stage, river discharge, flow velocity and sediment transport made during the test flow can be used together with a set of reach-averaged flow, suspended sand transport, and channel-edge sand-deposition models to generate a quantitative description of the response of the channel bottom and channel edge sand deposits in this fluvial system to high flows, and that this framework can be used to develop a very simple model for low frequency variations in sand movement between the channel bed and the channel edges.

Background

The Colorado River between the Glen Canyon Dam and Lake Mead is approximately 470 km long. It is deeply incised throughout most of its length, and its channel is subjected to rockfall along the margins and coarse sediment bearing floods and debris flows from steep tributaries. The consequence is a complex channel that can vary in depth from a few meters to more than ten meters on the downstream scale of a few channel widths. There is, however, no high-resolution map of the river corridor over most of its length. Direct measurements of flow, sediment transport, and erosion /deposition are sparse, and they often are strongly affected by local topography. In addition, the data sets typically are not comprehensive enough to generalize with confidence. To overcome these difficulties a theoretical framework in which to imbed old and new measurements is essential. A 470 km long reach of a complex riverine system simply can not be investigated productively without heavy interactive use of both empirical and theoretical scientific methods. To provide a framework for the measurements made during the research flows of 1990 and1991, a procedure called reach averaging was employed by Wiele and Smith (1996). A reach-averaging framework provided a basis for design of the flow and sediment transport measurement program for the test flood, and it forms the basis for the analysis of test flood results presented herein.

Need for Reach Averaging

In order to determine the flow field in a geometrically complicated system, such as the Colorado River system, topographically induced convective accelerations would need to be resolved, and to do this successfully the topography and the bed roughness field for the reach of interest would need to be well resolved. This amount of geometric information is rarely procured in most fluvial hydraulics studies and is not available for the Colorado River. An alternative approach is to average the channel geometry, the flow, and the suspended sediment transport fields over a long reach in order to suppress the topographically induced convective accelerations and the topographically induced variability in the suspended sediment flux field. If high resolution, river-corridor, topographic data is sparse, or if large scale (rather than local) information about a riverine system is desired, a reach-averaged approach to evaluating the flow and sediment transport in the system is the most effective procedure.

Hydraulically Similar Reaches

At the core of a reach-averaged framework is a satisfactory characterization of the average geometry of river segments that are very long relative to their widths and that have similar hydraulic and geomorphic properties. For the Colorado River, ten such reaches were established by Smith and Wiele (unpublished manuscript; see Griffin, 1997) in the 387 km long segment between the Glen Canyon Dam and Diamond Creek). These are denoted *hydraulically similar reaches*. The data on which the average channel shapes for discharges below 800 m^3/s in these reaches are based comes from a set of 199 cross-sections measured by Wilson (1986). Owing to the high degree of variability of the river-channel geometry on a downstream scale that is small relative to the approximate 1.6 km spacing of these cross-sections, each can be considered statistically independent of all others. These cross-sections, therefore, were grouped and averaged over the hydraulically similar reaches. Cross-sections for longer reaches can be obtained by combining those for the hydraulically similar reaches in an average weighted by the ratios of the lengths of the appropriate hydraulically similar reaches, or fractions thereof, to the length of the segment of the river under consideration. A typical long reach that is frequently used in this paper is from the Glen Canyon Dam to Diamond Creek. Models that use the cross-sections and other properties for a sequence of hydraulically similar reaches are denoted *linked-reach* models, models that use properties for a single long reach are called *single-reach* models, and the single-reach model for the Glen Canyon Dam to Diamond Creek is called the *Grand Canyon* model. The actual downstream distance that is used for a particular calculation depends on the purpose of the calculation, the density of data needed in the calculation, and amount of data available.

Average river corridor cross-sections for stages associated with discharges from 142 m^3/s to about 14,000 m^3/s (representing a reach-averaged stage range of 5.48 m to 25.8 m in the Grand Canyon cross-section) could be calculated for 9 very short reaches. These were used by Griffin (1997) to extended the reach-averaged cross-sections for all of the hydraulically similar reaches to stages above 8.90 m (from the Grand Canyon cross-section) associated with an 800m^3/s discharge. This was done by relating each high-stage cross-section with the river level rock type at that location, then grouping and averaging any cross-sections for reaches of the same rock type to produce estimated high-stage cross-section profiles. The 142 m^3/s river width from each high-stage profile was compared to the river width for the hydraulically similar reach in which the high-stage profile was located, to insure consistency. In a few cases, the high stage profiles had to be rescaled to the width of the hydraulically similar reach in which it was located. Griffin's calculation was necessary because long reaches of the river, bounded by rock types that produce a similar river-channel morphology, are devoid of the river-bank-resolving topographic information required for modeling high-stage flows and high-stage reach-averaged erosion/deposition. Griffin's cross-sections for the hydraulically similar reaches are the same as the ones based on the Wilson (1989) data for discharges below 142 m^3/s.

Reach Averaged Flow

Reach averaging is a precise mathematical procedure that has well defined fluid-mechanical and sediment-transport consequences. Reach-averaging is derived by averaging the governing equations over the appropriate downstream length. The effect on the flow velocity is to transform the unresolved flow accelerations into an enhanced effective roughness (Wiele and Smith, 1997). This has the advantage of permitting flow calculations to be done for reaches of complex, unresolved channel topography with relatively simple equations, but it means that the effective channel roughness must be determined empirically. In the Colorado River, the reaches are long enough (several tens of kilometers or more) that the reach-averaged river slope remains constant. In contrast, the local slope is composed of sequences of flatter sections over pools and steeper sections through rapids. As the discharge in the river increases, some of the rapids become submerged and the local slope steepens over the remaining pools. This causes the effective roughness to decrease faster with rising stage than would be the case for a gravel bed of fixed size. All of these effects are hidden in the stage-dependent effective roughness (called the *friction function* hereafter). Prior to the test flood there were no data with which to determine the friction function for any of the hydraulically similar reaches at discharges above about 850m^3/s.

Reach-Averaged Suspended Sediment Transport

For the suspended-sediment transport field, reach averaging produces a simple cross-sectional structure but at the price of making the sediment discharge knowable only empirically. As in the case for flow, the entire reach must act in the same manner, so the possibility of local erosion and deposition inside the reach must be suppressed. This means that for sediment of a given type (e.g. coarse silt, very fine sand, fine sand, medium sand etc.) the same amount of that material must pass through each cross-section.

The actual local flux of each type of suspended sediment depends primarily on the near-bed velocity (i.e. on the skin-friction shear velocity) and on the fraction of the bed covered by sediment of that type (Smith, 1977; Smith and McLean, 1977a, 1977b; Gelfenbaum and Smith, 1986; Kachel and Smith, 1989; McLean, 1992). Erosion of a particular type of sediment occurs when there is a divergence in the transport rate of that type of sediment, and deposition occurs when there is a convergence in this transport rate. Adjustment in area of the bed covered by a particular type of fine sediment occurs as follows. If the transport rate for a particular type of sediment (e.g. size) is higher into a location than out of it, there is a convergence in the sediment transport rate and deposition results. This occurs until the bed coverage of that material increases to the point where the efflux of that type of sediment equals its influx. Conversely, if the efflux originally exceeds the influx, there is a divergence in the transport rate, erosion occurs, and the bed coverage of that material is reduced until the transport rates become the same.

Except in a very few areas under abnormal circumstances (such as just downstream from the mouth of a sand transporting tributary right after a major flood on that tributary), the bed of the

Colorado River is composed of gravel or bedrock which is only partially covered by fine sediment of any type. In this situation, the primary mechanism by which a segment of a river attains the same transport rate through all cross-sections is by adjusting the area of the bed covered with fine sediment to produce the same downstream sediment flux everywhere. This adjustment in area occurs by the mechanism described in the previous paragraph, except that, in this case, all of the fine sediment should be thought of as a single type of material. This adjustment process occurred at all downstream length scales during the test flow

The time and space scales of the reach-averaged suspended-sediment field are connected more tightly than was the case with the flow field, because a redistribution of the sediment on the river bed by the mechanism described in the previous paragraph is required. In the case of the flow field, times that are considered to be resolved by a reach-averaged model must be long compared to the time it takes a fluid particle to pass through the reach. In the case of the suspended sediment field, the resolvable times must be long relative to the time it takes to rearrange the suspendable material, namely the fine sediment, on the riverbed. If a reach is short, then the sediment on the bed can be rearranged much faster than if it were long. The internal adjustments by which this rearrangement occurs depend on the bed topography and the surface areas and depths of the patches of fine sediment inside the reach; therefore, in reaches for which the bed topography and distribution of fine sediment are undetermined, the reach-averaged percent coverage of the bed (or similarly the reach-averaged near-bed concentration and, thus, the suspended-load discharge) for sediment of various sizes can be determined only empirically. Adjustment times increase with reach length because more sediment has to be moved for greater distances at larger downstream length scales. For reach-averaged, suspended-load transport modeling in the Colorado River, it is essential to know the adjustment time for the entire system or at least a large segment of it. To do this theoretically, the detailed geometry of the river channel, the sizes and shapes of the pockets between the pebbles, cobbles and boulders on the riverbed, and the initial amounts and locations of all fine sediment on the riverbed must be known. For a long, complex system, procuring this information is not practical. However, the riverbed adjustment time scale can be easily determined from measurements made during the test flow. The time scale for the fine sediment on the river bed to come to equilibrium during the test flow for a 170 km reach of the Colorado River is estimated in the section below denoted "Other Test Flood Results".

Reach-Averaged Deposition

The assumption that the system has come to equilibrium also means that all of the previously open depositional environments in the reach have been filled with fine sediment. The rate at which this occurs can be evaluated by applying reach-averaging methods to conditions from before the test flood and using stage and discharge data from during that flood. If all of the available depositional surfaces are covered with sand to river level, with an angle of repose slope (approximately 30°) on all riverward sides, then the reach-average cross-section is considered to be full. The pre test-flood state can be estimated using the reach-averaged cross-section of the river corridor (single reach) developed by Griffin (1997). The reach-averaged system with all of the potential depositional sites full, as defined above, then can be added to this cross-section. The difference between the original and final geometries yields the volume of the test-flood deposit.

Use of Reach-Averaged Properties to Calculate Local Flows

Although not done in this paper, cross-sectionally averaged velocities predicted by a reach-averaged flow model can be used to obtain local velocity fields at any site along the river, as long as the topography and bed roughness patterns are known over a reach that is several tens of river widths in extent. For a long, horizontally uniform reach this could be done using a generalized version of the logarithmic velocity structure, but in most locations it likely to be necessary to use a step-backwater calculation that resolves cross-sectionally averaged, topographically-produced convective accelerations. For even greater accuracy, a multi-dimensional model could be used to obtain the velocity field (for example, see Wiele et. al.,1997), but such models require a much greater density of bed morphology and bed roughness information to yield results that are equivalent or better than those produced by simpler models. As model complexity increases, the amount of information necessary to be input increases many fold. Reach-averaged suspended-sand fluxes also can be used to support local sand-transport calculations. The reach-averaged model routes the water and suspended sediment down the river, into and out of the location of interest. The local model redistributes the flow and suspended sand inside the shorter reach, commensurate with the constraints imposed by the topographic and bed roughness fields, while accommodating the imposed discharges of water and sand.

Improvement of the Discharge Routing Model Using Test Flood Data

An accurate flow model is required to calculate discharge as a function of time and location in the Colorado River through the Grand Canyon. This discharge information is required by all flow, suspended sediment, and erosion/deposition models. It also is needed in many local physical, chemical, and biological studies in and along the river. The diurnal discharge wave produced by normal operation of the Glen Canyon Dam typically is over 100 km in length, and, thus, averages the topography of the river channel on the scale of tens of kilometers. This makes a reach-averaged approach ideally suited to routing diurnal and lower frequency discharge waves down the Colorado River from the Glen Canyon Dam to Lake Mead.

Previous Work

A kinematic wave model (Lighthill and Whitham, 1955) developed by Wiele and Smith (1996) was used prior to the test flood by Wiele (1996) to translate and deform the then proposed spring 1996 test flood discharge as it propagated downstream from the Glen Canyon Dam. Owing to the high but unresolved relative roughness of the system and the fact that a stage dependent part of the elevation drop in the river occurs through rapids, an empirical expression for the friction function was developed and used by Wiele and Smith (1996). The friction function was constructed using stage-discharge information at near power-plant-capacity procured by Wilson (1986), dye advection information obtained during a steady 425m^3/s research flow in 1991 (Graf,1995), and wave travel time information from the 142m^3/s to 425m^3/s research flow B in 1991. Until the test flood, there was no information available for above-power-plant-capacity flows that could be used to extend the friction function to high stages, so it was extrapolated to the 1275 m^3/s test flood level by Wiele (1996). After the test flood, however, Wiele and Griffin (1997) used the falling limb of the test flood hydrograph to develop a friction function that produces considerably more accurate wave travel times at high discharges. Also, dye advection velocities at a discharge of 1280m^3/s were measured during the test flood and were used to test the new friction function. Wilson's cross-sections extended only to the river surface at discharges that varied around 800m^3/s. Therefore, Wiele (1996) and Wiele and Griffin (1997) use extrapolated high-stage, reach-averaged channel cross-sections. Owing to the empirical nature of the friction function, small errors in reach-averaged cross-section shape are folded into it.

This paper begins with the Wiele and Smith (1996) model, as updated by Wiele and Griffin (1997), but uses the channel geometry developed by Griffin (1997). The goal is first to generate a model that will produce very accurate discharge wave hydrographs with shapes undisturbed by suspended sediment transport and deposition, then to compare these hydrographs to measured ones at locations downstream of Bright Angle Creek where the discharge wave is no longer evolving. The mean discrepancy between these hydrographs then provides a means of calculating sediment deposition rates.

Single Reach Model

The friction function is the ratio of the cross-sectionally averaged, then reach-averaged flow velocity to the reach-averaged shear velocity. The reach averaged shear velocity is the square root of the product of the reach-averaged slope, the reach-averaged hydraulic radius (approximately the average depth), and the acceleration due to gravity. Using the cross-section of Griffin (1997), and the method of Wiele and Griffin (1997) to calculate the friction function from the falling limb of the test flow hydrograph, results in the dashed line on figure 1. The large dots on figure 1 indicate the friction values for which the original routing model was calibrated, plus the one from the test-flood dye study (the latter being at the highest stage). Of these, only the value from the $425 m^3/s$ research flow was used in the single-reach (Grand Canyon) model developed for this paper. In figure 2, hydrographs calculated using this friction function are compared with measurements at Diamond Creek. The differences between the measurements (dots) and the calculated hydrographs, shown in figure 2a, represent an improvement of over an hour in modeled wave arrival time at Diamond Creek relative to the results of Wiele (1996). The average error in time in this new single reach model is 19 minutes on the rising limb and 24 minutes on the falling limb at Diamond Creek. The method of Wiele and Griffin (1997) uses the difference in measured falling limb hydrographs at Lees Ferry and Diamond Creek. The Lees Ferry hydrograph is back-calculated to the Dam, then used to drive the model that is used to calculate the dashed lines in figure 2, so if the inversion to obtain the friction function were perfect, the calculations (represented by the dashed line in figure 2b) and the measurements (represented by dots that are so dense that they appear to trace a solid line) would be identical. The small discrepancy between them provides a measure of computational error, probably associated with smoothing the friction function, to obtain the dashed line in figure1.

Linked-Reach Model

During the research flows of 1990 and1991, temporary stage gages were placed approximately every 8 km along the river between the dam and Lake Mead. Twenty-nine of the original fifty instruments were still in operation at the time of the test flow, although two did not record data for the entire period and four behaved erratically. Reach-averaged values of the flow velocity were obtained from a dye study designed for that purpose for the test flood (Konieczki et. al.,1997) and from a dye study conducted during the 425m^3/s research flow (Graf, 1995). The two dye studies were then used to relate each of the traces for the rising and falling limbs of the test flood hydrographs from the remaining set of temporary stage gages. These were related to the average cross-section of Griffin (1997) for the hydraulically similar reach in which it was located using the method described by Griffin et al (in review). Once an accurate, reach-averaged channel shape and an accurate stage-discharge relationship was available for each of these hydraulically similar reaches, an area-discharge relation was calculated and used to generate a friction function for each hydraulically similar reach. This permitted any discharge to be routed down-river reach-by-reach, greatly improving the accuracy of the routing algorithm, particularly upstream of Bright Angel Creek.

Linked-Reach Model with Topographically Caused Convective Accelerations

The rising limb of the flood hydrograph also has been used by the authors to improve further the reach-averaged kinematic wave model. On the falling limb of a hydrograph, the surface slope due to the wave decreases as the wave propagates downstream and becomes very small relative to the bed slope after which this part of the wave propagates as a purely kinematic feature, meaning that for a given channel geometry the wave velocity depends only on stage (which sets the cross-sectional area) and bed roughness. Therefore, distortion of the hydrograph directly determines the bed roughness as it varies with stage (i.e. sets the friction function). On the rising limb, in contrast, the faster moving crest of the wave causes the wave to become steeper, and this means that flow due to the steep wave front eventually must be taken into account. In a uniform channel this is easily done (see Lighthill and Whitham, 1955) and was included by Wiele and Smith (1996), but accelerations caused by variations in cross-section of the Colorado River are of a scale that produces a further nonlinear contribution to the generalized kinematic wave theory after reach-averaging. The rising limb of the test flow showed that this effect could not be ignored and also provided a means of estimating it in reaches for which the bed topography is unknown, namely most of the river. Fortunately, in the reaches near the mouth of the Little Colorado River where the river is wide and this nonlinear effect is greatest, there is enough topographic information to calculate its magnitude and demonstrate its connection to the anomalous behavior of the rising limb of the generalized kinematic wave. In the other reaches, the topographic irregularities that produce this effect can not be resolved, but they can be back-calculated from the shape of the rising limb of the locally measured test-flood hydrograph. Once estimated, however, the correction can be applied to all hydrographs because it is topographically and stage based. Figure 3 is a comparison of the calculated and measured rising limbs of the test-flood hydrographs at the gage above the Little Colorado River. Although going from the single reach to the linked reach model yields the greatest benefit, including this nonlinear effect further improves the fit of the calculated to the measured hydrograph.

Computation of Reach-Averaged Deposition Rates

Hydrographs from all of the temporary stage gages in operation during the test flow were converted from stage to discharge using the method described above. These then were compared to hydrographs calculated using the linked-reach flow model described in the previous section. Below Bright Angel Creek, the discharge wave was no longer evolving in time and the large scale convective accelerations that caused errors on the rising limb of the hydrograph near the Little Colorado River were more subdued and could be neglected to a first approximation. Therefore, the model should have predicted the measured hydrographs here very accurately. Instead, the hydrographs derived from measurements made at the temporary stage gages during the first half day of the test flow were everywhere lower than the ones obtained from the flow model in that part of the river. A comparison of a pair of hydrographs is shown in figure 4 for the temporary stage gage 154 km downstream of Lees Ferry. In this figure, a small difference in phase between these two curves has been removed. The dots in the figure are from measurements made at the temporary stage gage and the dashed line is from the linked-reach discharge routing model. The discrepancy displayed in this example is essentially the same as the average discrepancy between Bright Angel Creek and Diamond Creek (shown in figure 5). This solid line in figure 5 graphs with time the average discrepancy between the discharge hydrograph computed with the flow model and the one computed from the output of the temporary stage gage for the reach between Bright Angel Creek and Diamond Creek. The fact that the model hydrograph reaches the asymptote sooner than the one derived more directly from measurements, and that the discharge at the dam was constant throughout the period of interest, suggests that somehow water is being lost from the test flood to the channel margins from the test flow. Water is being lost to the channel margins both by ground water flow into the banks and through direct deposition in the interstices of the sand. Simple estimates using Darcy's law together with permeabilities for mixtures of fine and medium sand, however, demonstrate that the flux of water into the sand deposit is much too high to be the result of this mechanism.

Alternatively, as sand is deposited from the moving fluid, water is trapped in the interstices of the sand grains and is lost from the flow, the boundary of the flow being the edge of the non-moving material. Normally packed sand on the margins of the Colorado River has a porosity of approximately 0.33; therefore, we can multiply the mean discharge discrepancy curve by a factor of three and convert it conservatively to a sand deposition rate graph. The solid line in figure 5, therefore, can be considered the reach averaged water deposition rate as a function of time for the reach from Bright Angel Creek to Diamond Creek, and the dashed line in this figure is the associated minimum sand deposition rate for this reach. The dotted line on figure 5 represents the average sand deposition rate for the reach from the Lees Ferry to Diamond Creek, computed in the manner described in the following paragraph.

To test whether or not the sand deposition rate curve gives a reasonable deposit volume, the area under it can be compared to the deposit volume per unit river length determined from a simple reach-averaged, equilibrium-deposition model. Using the reach-averaged cross-section from Bright Angel Creek to Diamond Creek and filling all of the area below the river surface with newly deposited sand to a subaqueous angle of repose of 30°, as suggested in the section on reach averaging, gives a mean deposit of 18 m^3/m (cubic meters per meter of channel length). The area under the average sand deposition rate curve for the same reach (i.e. the area under the dashed line on figure 5) is 13 m^3/m.. Taking these numbers at face value, indicates that the available depositional sites in the reach are, on the average, 72% full. The areas under the sand-deposition-rate curves for the individual temporary stage gages, however, vary by more than a factor of two and the standard deviation for this population is 9.85 m^3/m. Moreover, the cross-section-based deposition model results are likely to have errors of 20% or more, so the computed values of 13 m^3/m and 18 m^3/m are not significantly different. The depositional rate calculation shows that the depositional sites between Bright Angel Creek and Diamond Creek were at least 72% and possibly completely full after the first 13 hours of the test flow. The available depositional area between Lees Ferry and Diamond Creek is 25 m^3/m. The ratio of the two deposit volumes per unit river length determined using the equilibrium reach-averaged deposition model is 1.4. Scaling up the sand deposition rate curve for the shorter Bright Angel to Diamond Creek reach (dashed line in figure 5) to the longer Lees Ferry to Diamond Creek reach using this ratio gives the dotted line in figure 5. The area under the dotted line in figure 5 is 18m^3/m. The deposition is greater in the longer reach, because it has a broader cross-section, produced in large measure by the open area between Saddle Canyon and the beginning of the Granite Gorge.

Figure 6 shows the average river edge profile for the reach between the Lees Ferry and Diamond Creek (heavy solid line). Included on it is the stage of the test flow. Also shown on figure 6 are the sand deposits (stippled area) predicted by the simple deposition model described in the section on reach averaging and used in the previous paragraph. The angle of repose is a static entity, so it is independent of the fluid medium. If, however the fluid medium is more dense on the inside of the deposit (e.g. water) than on the outside (e.g. air), there is an outward directed pore pressure and the effective angle of repose is reduced substantially. According to Budhu and Contractor (1991), a typical equilibrium value for a sand deposit having been subjected to bank storage from a river stage that oscillates between two levels with a diurnal period is about 11° .This value can be used in the simple equilibrium deposition model by assuming that all newly deposited sand will eventually be eroded to a surface with an 11° slope that extends from the intersection of the submerged bank and the low stage level to the high stage level of the oscillatory discharge. At the intersection of the 11° slope and the high stage level there is no pore pressure, and the angle of repose returns to 30°, or the 11° slope intersects a non-erodable surface. In the latter case, the 30° slope begins again at the intersection of erodable surface and the high stage of the oscillatory flow. After the test flow, the discharge oscillated in this manner between 464 m^3/m and 552 m^3/m for several months. This period of time is long enough for this equilibrium model to give reasonable results. As a consequence of the oscillation in stage in the months following the test flood, it appears that about 30% of the test flood deposit was returned to the bottom of the river (figure 7). Right after the high flow, the discharge was dropped to 225 m^3/s, which produced a stage of 6.16 m, but it remained at this level for only a few days. A substantial fraction of the post-test flood erosion may have occurred during this event, but the reach-averaged model cannot be used in this transient situation. In this transient case, the water in the bank drains before the effects of the pore pressure disappear as the water table drops.

Other test flood results

Flow velocity and suspended sediment concentrations were measured at three locations on the Colorado River during the test flood. These were just upstream of the mouths of (1) the Little Colorado River, (2) Bright Angel Creek, and (3) National Canyon. (For the locations of these cableways see figure 1 of Patten et. al., this volume.) The flow, suspended sediment, and percent silt and clay data for all three sites were collected by the USGS and are tabulated by Konieczki et. al.(1997). Different types of measurements were emphasized at each of the three sites, but suspended silt-plus-clay and suspended sand discharges could be determined from all three sets of data. Furthermore, near bottom silt-plus-clay and near bottom sand concentrations could be determined from the National Canyon data set. These measurements are used in the analysis described in the following paragraphs.

Secondary Circulation

The response of the National Canyon site to the test flow was observed in detail by Smith. The flow field at this location showed a distinct low velocity zone at the surface near the both edges of the river, which is indicative of a strong secondary circulation with flow toward the river center near the surface and flow toward the banks near the river bed. The suspended sand concentration profiles were normal near the center of the channel, but were very steep and of low magnitude near the river margins, which is consistent with strong vertical velocities associated with an energetic river-edge boil field. From the banks, a boat, and the cableway, an energetic river-edge boil field was obvious, as was a strong but highly sinuous convergence zone near the center of the river along which all of the debris lined up. Velocity measurements at the upstream Little Colorado and Bright Angel Creek cableways were insufficient both horizontally and vertically to resolve any cross-sectional circulation, but video tapes taken during the test flood at other locations along the River show similar surface features, indicating that the secondary circulation probably was a widespread phenomenon. It is likely that this strong secondary circulation was responsible, at least in part, for the rapid, reach-averaged deposition on the channel margins discussed in the previous section.

Near-bed Suspended-Sediment Concentrations

Near-bed silt-plus-clay and suspended sand concentrations as functions of time in days after the beginning of the steady high flow are shown in figure 8. These have been normalized by their respective values on the sixth day of high flow to show that after the second day, the ratio of sand to silt-plus-clay (primarily coarse and medium silt) remained constant at a ratio of 13.5. In contrast, during the first day the riverbed contained 70% more silt than it did after it came to equilibrium, and the sand to silt-plus-clay ratio was only 8.4. This constancy of near-bed concentrations suggests that the composition of the bed surface had ceased evolving in time by the end of the second day. It also provides a mechanism for increasing the accuracy of the analysis of suspended sand concentration data from the test flood. Near-bed sand concentrations are difficult to measure accurately because of their sensitivity to local bed topography, bed coverage with fine sediment, and the near-bed structure of the turbulent flow. Silt concentrations are significantly less sensitive to all of these complications and provide a much better estimate of the cross-sectionally averaged, near-bed concentration field. Therefore, at locations where the cross-sectional structure of the sand concentration field has not been well resolved, the silt-plus-clay data (mostly silt in the Colorado River through the Grand Canyon) can be used the to get the temporal structure of the cross-sectionally-averaged, near-bed, suspended sand concentration and a sand concentration model can be used to calculate the cross-sectional structure of the sand concentration or sand flux fields. The available sand concentration data then can be regressed on this structure to obtain actual values.

Cross-Sectionally Averaged Suspended Sediment Concentrations

For the case of the sand concentration field at the above National Canyon site, the good agreement between the two sets of data in figure 8 indicates that the cross-sectionally-averaged, suspended-sand concentration field is likely to have been well resolved at this site. Consequently, it will be compared to data from the two other locations. Figure 9a displays such a comparison. It also compares silt-plus-clay concentration data from all three sites. Except for the National Canyon data, which was analyzed by the first author, the data used in this figure are from Konieczki et. al. (1997). At the above Bright Angel Creek site (denoted the Grand Canyon site by the USGS and in figure 9), concentration data were procured using two different methods (Konieczki et. al., 1997). Both data sets are shown in figure 9. The cross-sectionally-averaged concentrations denoted as Grand Canyon 1 and Little Colorado River were procured using standard USGS methods (Konieczki et. al., 1997) and the information denoted National Canyon was calculated from a dense set of point measurements of suspended sediment concentration. The data denoted Grand Canyon 2 is from a cross-sectionally sparse set of point samples.

In order to increase the accuracy of the point values of sand concentration at the National Canyon site for more detailed spatial analysis, they were filtered using a two point running mean. It is these filtered data that were averaged across the cross-section and that are shown in figure 9a after the first day. The single value, calculated for the first day is also shown. The standard deviation for the filtered data set is 8%. Except for the spatially under-sampled data set (Grand Canyon 2), both the sand and silt-plus-clay data show a remarkable consistency with the structures displayed by the more accurate National Canyon curves. The silt-plus-clay concentrations are the same at all locations after the third day of the test flood and the cross-sectionally averaged sand concentrations for the two down-river locations (Grand Canyon and National Canyon) are the same to well within measurement error throughout the entire period for which there is data. The cross-sectionally averaged sand concentrations for the Little Colorado River site are half the values for the downstream sites, indicating that the Little Colorado River is contributing about half of the sand; while the other half is coming down Marble Canyon. Temporal structures of the sand concentration decreases for National Canyon data and for the two data sets procured using standard USGS methods are shown normalized by their day 6 values in figure 10. The ratios of the normalizing factors are 0.93 and 0.49 relative to the National Canyon value. Standard deviations for the two upstream data sets around the curve for National Canyon are both less than 12%.

Discussion of Test Flood Results

The approach to evaluation of test flood data for the purpose of determining the response of the sand in the Colorado river to a high flow is iterative. The basic component of the work uses comparisons between a reach-averaged flow model and stage and discharge measurements to successively improve the flow model until the discrepancy between the measured and calculated hydrographs in the canyon, between Bright Angel Creek and Diamond Creek, can be attributed to loss of water to the channel margins. This loss could be a result either of water transport into the banks by ground water flow processes or direct deposition of water with the sediment, the boundary of the river being at the tol of the new deposit. Both processes were occurring during the test flow, but the period of the water loss combined with the low permeability of the sand deposits precludes that alternative. Moreover, the newly deposited sand was already wet reducing its capacity to remove water directly from the river.

The reach-averaged rate of water deposition was converted to sand deposition using an estimated porosity, then the sand deposition curve was integrated to give the total estimated deposition per unit river length between Bright Angel Creek and Diamond Creek. A simple reach averaged deposition model using the topography generated by Griffin (1997) then was used to calculate the deposit volume per unit length of river, first for the reach between Bright Angel Creek, and then for the one between Lees Ferry and Diamond Creek. Comparison of the deposition calculated from the deposition rate curve and that from the deposition model indicated that 70& of the sand was deposited in the first 13 hr of the test flood. Discrepancies between the modeled hydrographs and those measured at specific sites were highly variable and the difference between the two calculations probably is not significant. The ratio of the results from the deposition models for two reaches was then used to rescale the deposition rate curve for the longer Lees Ferry to Diamond Creek reach. Owing to the absence of a high-resolution map of the river corridor the actual reach-averaged cross-section is not known. The one estimated by Griffin (1997) probably leads to at least a 20% error in deposit volume per unit river length.

Conclusions

The test flood provided an opportunity to test existing hypotheses about how the flow and sediment transport systems operated on the local, reach, and large segment scale in the Colorado River through the Grand Canyon. Our hypotheses on the basic processes and their interplay proved to be correct, but our hypotheses with respect to the rates at which sediment is eroded from the river bed and deposited on the channel margins proved to be too pessimistic. Data from the test flood indicated that at discharges of 1200 m^3/s or higher, enough sediment could be moved from the bed of the river to its edges in a 12 hour period to fill most of the open depositional sites.

Reach averaging produced an accurate flow model when data were available (1) to develop an accurate reach-averaged cross-sectional shape for characteristic segments of the river corridor, (2) to determine an appropriate stage-dependent roughness function, and (3) to determine the reach averaged, near-bed concentration of the suspended sand. Also, a simple reach-averaged model for sand deposition was shown to produce reasonable estimates of sand deposition.

Acknowledgments

The 1996 test flow was a huge undertaking, and the information on which this paper is based was obtained as the result of an intense effort on the part of a large number of very devoted scientists. To all of them we express our profound gratitude. Of special note is the terrific USGS crew that assisted the first author at the National Canyon site during the test flood, and particularly Kent Sherman, Rick Seiderman, and Jeff Phillips without whose commitment to the procurement of data of the highest quality, an important part of the test flood experiment would not have been successful, and this paper would not have been possible. We also want to thank Ned Andrews, Julie Graf, Mary Hill, David Topping, Kirk Vincent, and Steve Wiele for assistance and for numerous valuable discussions about the test flood. Special thanks goes to Bob Gauger for operation of the temporary stage gage network in a quality manner.

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Figure Captions

Figure 1.Comparison of calculated friction coefficient β as a function of stage to that obtained from dye advection and other measurements. The calculated friction coefficient is for the reach averaged flow between the Glen Canyon Dam and Diamond Creek using the mean channel geometry for this reach determined by Griffin (1997). Also used is the difference in falling limb hydrographs for the test flood at Lees Ferry and Diamond Creek. The abnormally rapid increase in β, that is, in the cross-sectionally averaged flow velocity relative to the shear velocity, as a function of stage, results from the high relative roughness and numerous elevation drops through rapids in the system.

Figure 2. Comparison of measured and calculated hydrographs at Diamond Creek for the test flood. The leading and trailing edges are presented in (a) and (b) respectively. The dots represent discharges determined in the standard USGS manner from measured values of river stage. The dashed line denotes a model-calculated hydrograph using the empirical friction function of figure 1. On the falling limb of the hydrograph the dots are so close together that they appear to merge into a solid line.

Figure 3. Comparisons of the measured hydrograph for the rising limb of the test flood at the gage above the Little Colorado River to ones calculated using various flow models. The dots denote discharges calculated in the standard manner from measurements of river stage. The heavy dashed line results from using a linked-reach model with a correction for depth variation caused

convective accelerations. The thin dashed line denotes the output from a linked-reach model without any depth variation correction and the light dotted curve is from a calculation using a single reach model.

Figure 4. Comparison of the hydrograph calculated using a linked -reach model (dashed line) and discharges determined from stage measurements (dots) for a site 154 km downstream of Lees Ferry. The difference between the two curves is typical of all comparisons of model-calculated discharges to ones determined from measurements made at temporary stage gages on the Colorado River between Bright Angel Creek and Diamond Creek. The difference displayed in this figure is very close to the mean discrepancy for the entire reach.

Figure 5. Average rates of water (solid line) and sand (dashed line) deposition as functions of time for the Colorado River between Bright Angel Creek and Diamond Creek during the test flow. Also shown is the average deposition rate of sand between Lees Ferry and Diamond Creek as determined by rescaling the dashed line by the ratio of reach-averaged deposit volumes per unit river length for the longer to the shorter reach.

Figure 6. Model estimated pre and post test flood reach-averaged channel shape. Deposition is assumed to occur until a 30° slope is produced along the channel margins. The gray area denotes the maximum amount of sand deposition that could have occurred during the test flood, given that the reach-averaged river corridor geometry has been calculated accurately.

Figure 7. Model estimated deposit volume remaining two months after the end of the test flow. The model uses an 11° slope for the surface produced by the pore pressure resulting from stage changes associated with diurnal discharge oscillations from 464 m^3/s to 552 m^3/s.

Figure 8. Cross-sectionally averaged concentrations of sand, and silt-plus-clay, at the cableway just above National Canyon, normalized by their values on day 6 of the test flood and graphed as a functions of time from the beginning of the high flow. It can be seen that the day 6 values are getting very close to the asymptotes delineated by each of the data sets. By the second day of high flow, the normalized sand and silt-plus-clay values are almost the same, and from day 3 on they are indistinguishable. This suggests that the composition of the riverbed has stabilized in this reach by day 3 , and further adjustments in time occur by reducing the fraction of bed covered by fine sediment rather than changing the relative concentrations of the various sizes comprising the distribution.

Figure 9. Comparison of cross-sectionally averaged sand concentrations (a), and silt-plus clay concentrations (b) at three locations along the Colorado River during the test flood. The lines connect the measurements from the gage site just upstream of National Canyon and the one just upstream of the Little Colorado River. The silt-plus-clay data from the three locations are somewhat dispersed in the early part of the test flood, but they appear to converge by day 4. The silt-plus-clay concentration at the gage upstream of the Little Colorado River was a factor of three lower than that for the other sites on day 1, but was only 25% lower by day 2 and only 15% lower by day 3. The sand data display a different pattern. In this case, the concentrations for

above National and above Grand Canyon gages are the same within the error bars of the latter data , but the measurements from above the Little Colorado River are only half that value.

Figure10. . Comparison of normalized, cross-sectionally averaged sand concentrations from the cableway upstream of National Canyon to those from the cableways upstream of Bright Angel Creek and the Little Colorado River. The Little Colorado, Grand Canyon-1, and National data sets all fall on the same curve indicating that a 170 km segment was adjusting similarly in time. The normalizing factors relative to the one for the National Canyon data set are 0.93 for the Grand Canyon 1 data and 0.49 for the Little Colorado River data.

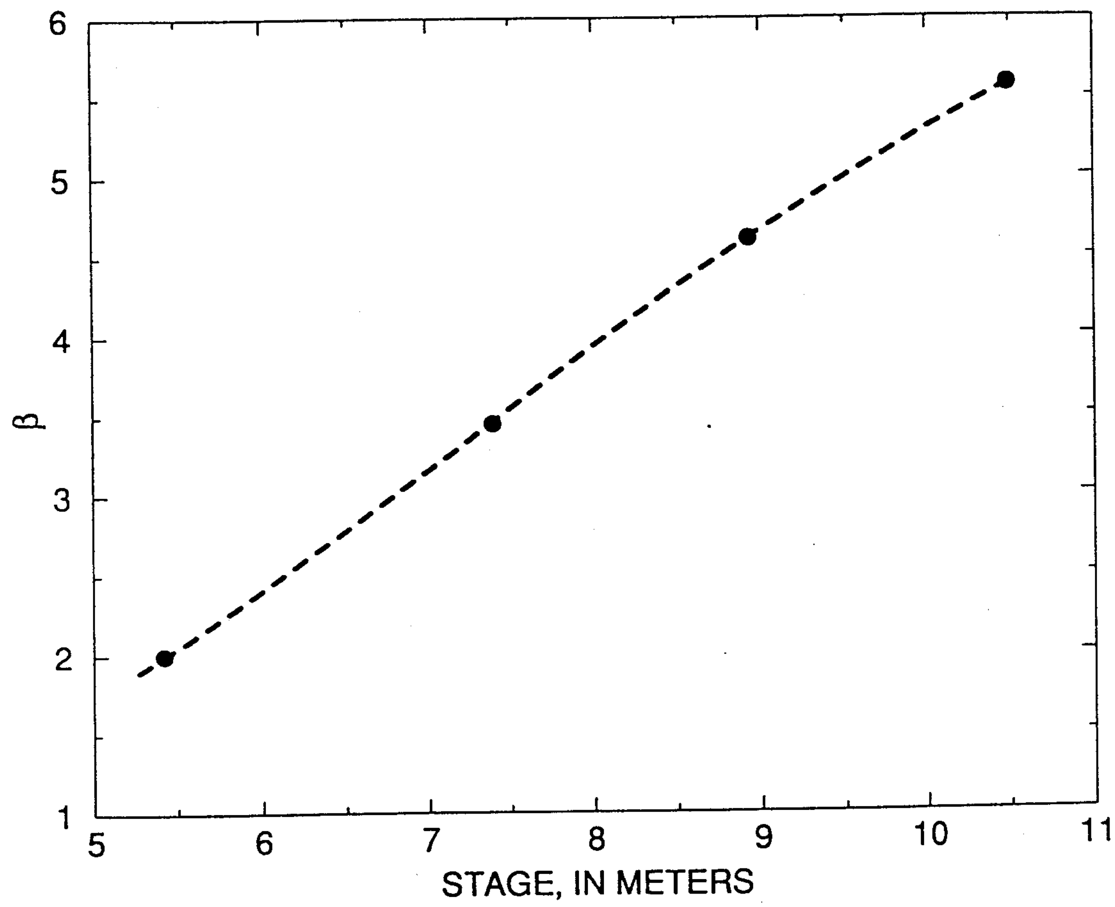


Fig 1

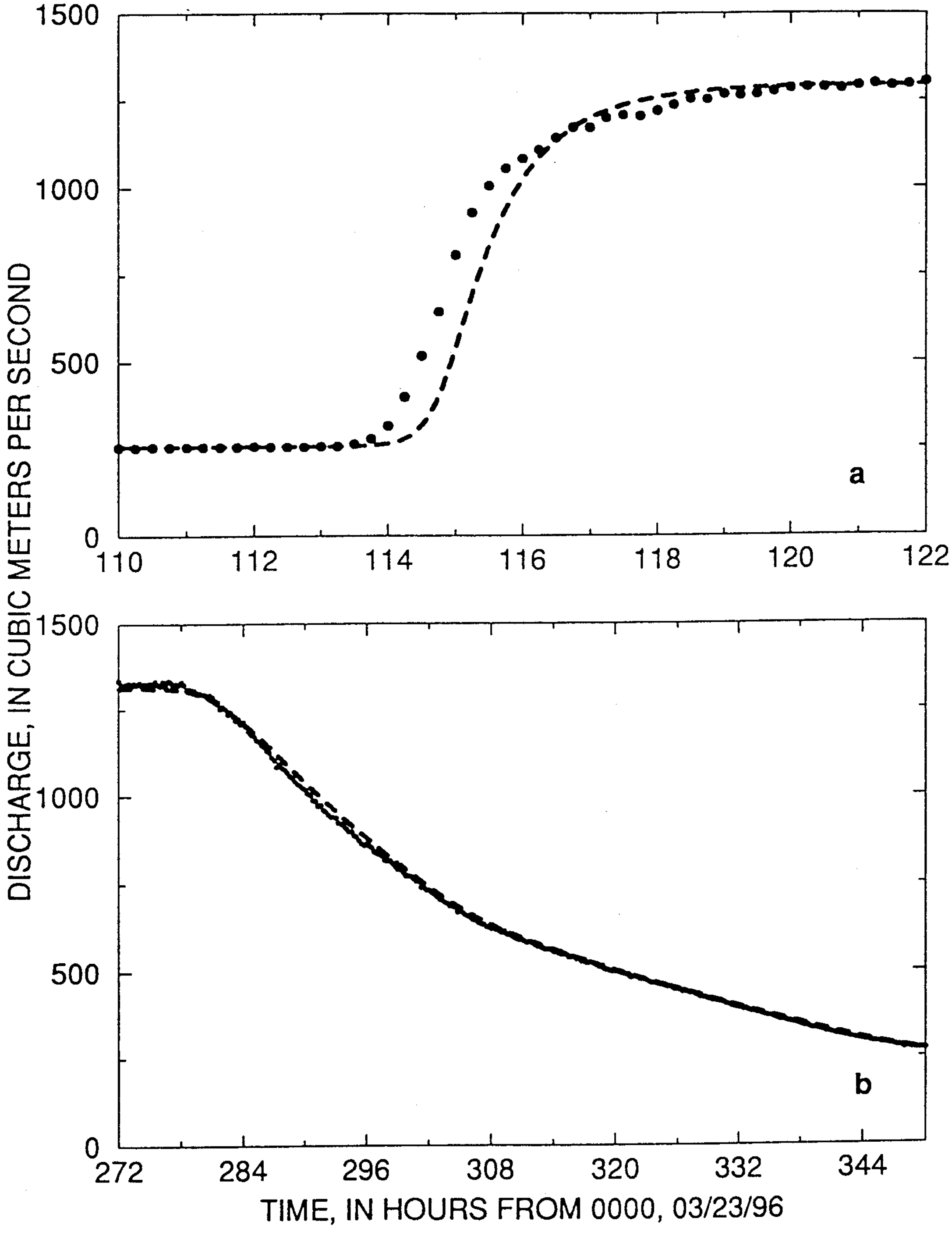

DISCHARGE, IN CUBIC METERS PER SECOND
1500
1000
500
0
110
112
114
116
118
120
122
a
1500
1000
500
0
272
284
296
308
320
332
344
b
TIME, IN HOURS FROM 0000, 03/23/96

Fig 2

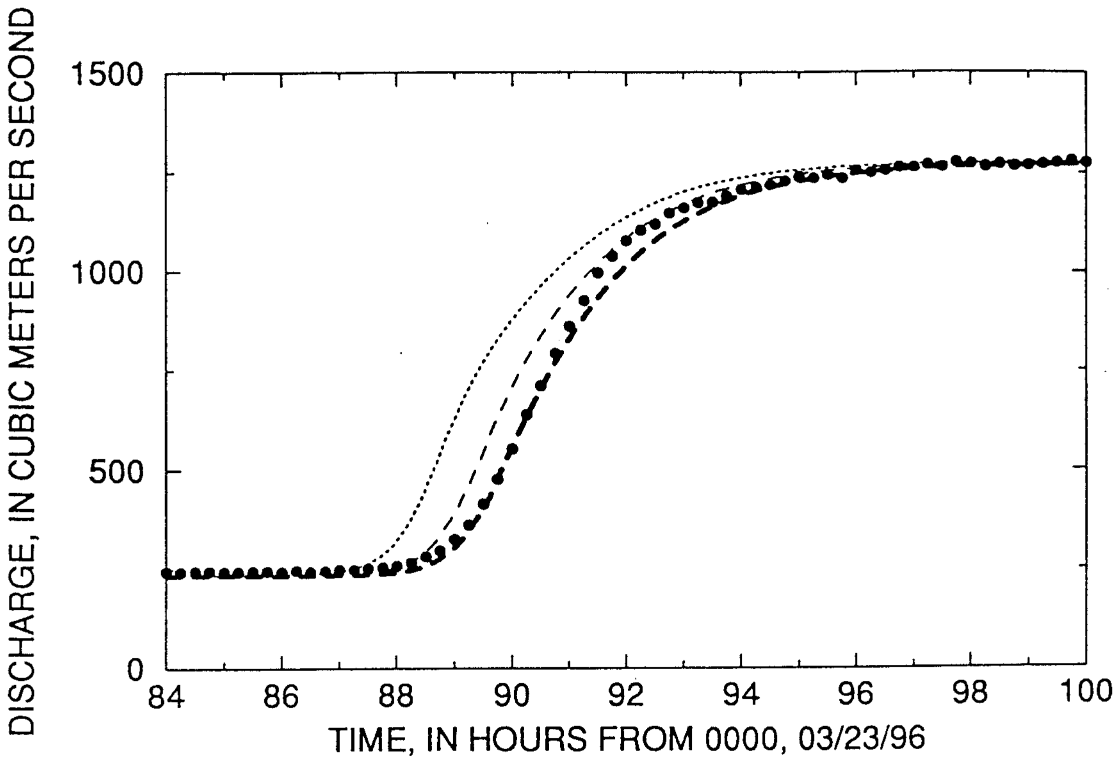
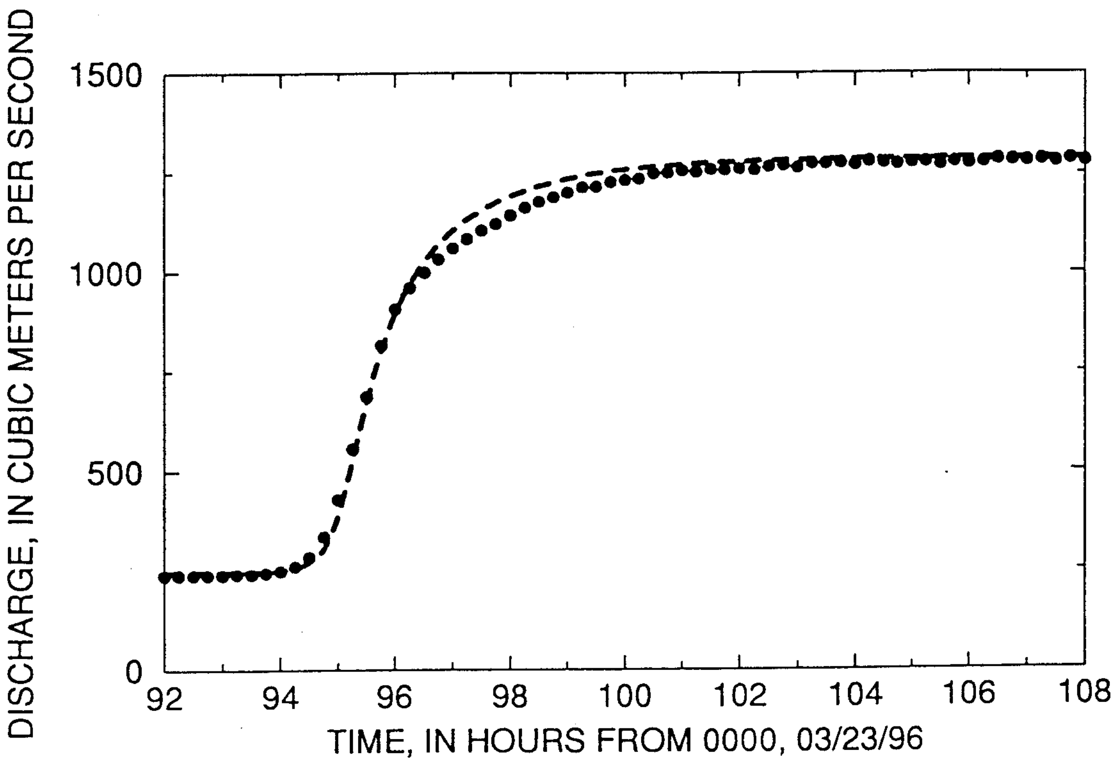

DISCHARGE, IN CUBIC METERS PER SECOND
1500
1000
500
0
84
86
88
90
92
94
96
98
100
TIME, IN HOURS FROM 0000, 03/23/96

Fig 3



DISCHARGE, IN CUBIC METERS PER SECOND
1500
1000
500
0
92
94
96
98
100
102
104
106
108
TIME, IN HOURS FROM 0000, 03/23/96

Fig 4

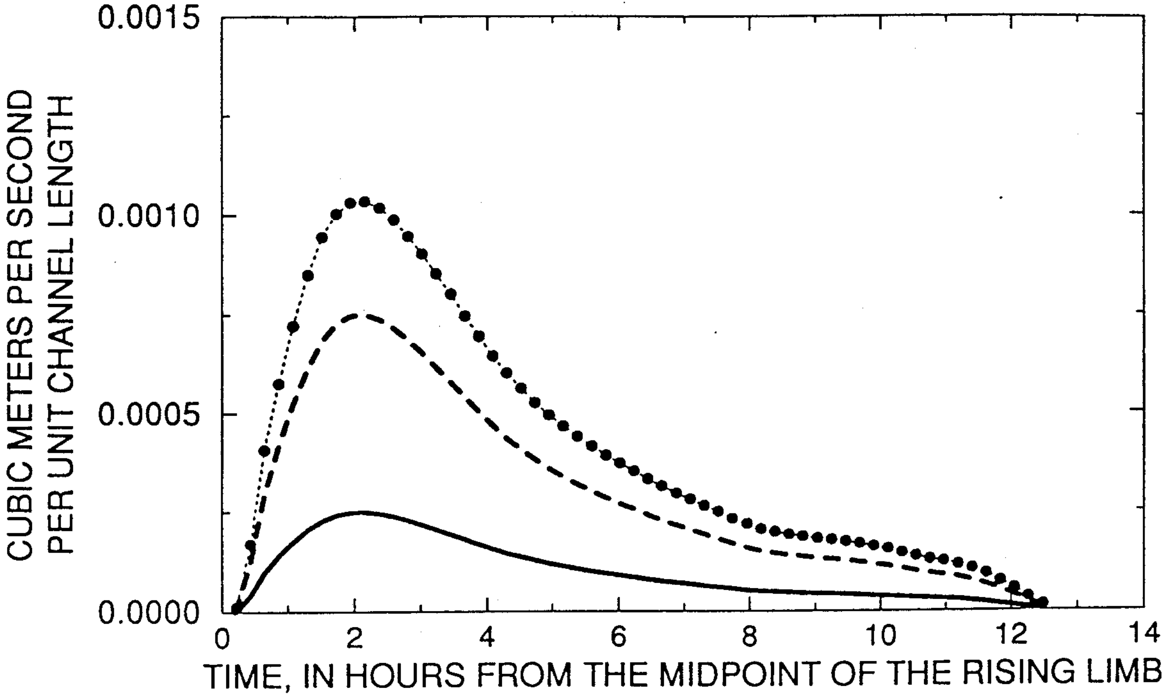

0.0015
0.0010
0.0005
0.0000
CUBIC METERS PER SECOND
PER UNIT CHANNEL LENGTH
0
2
4
6
8
10
12
14
TIME, IN HOURS FROM THE MIDPOINT OF THE RISING LIMB

Fig 5

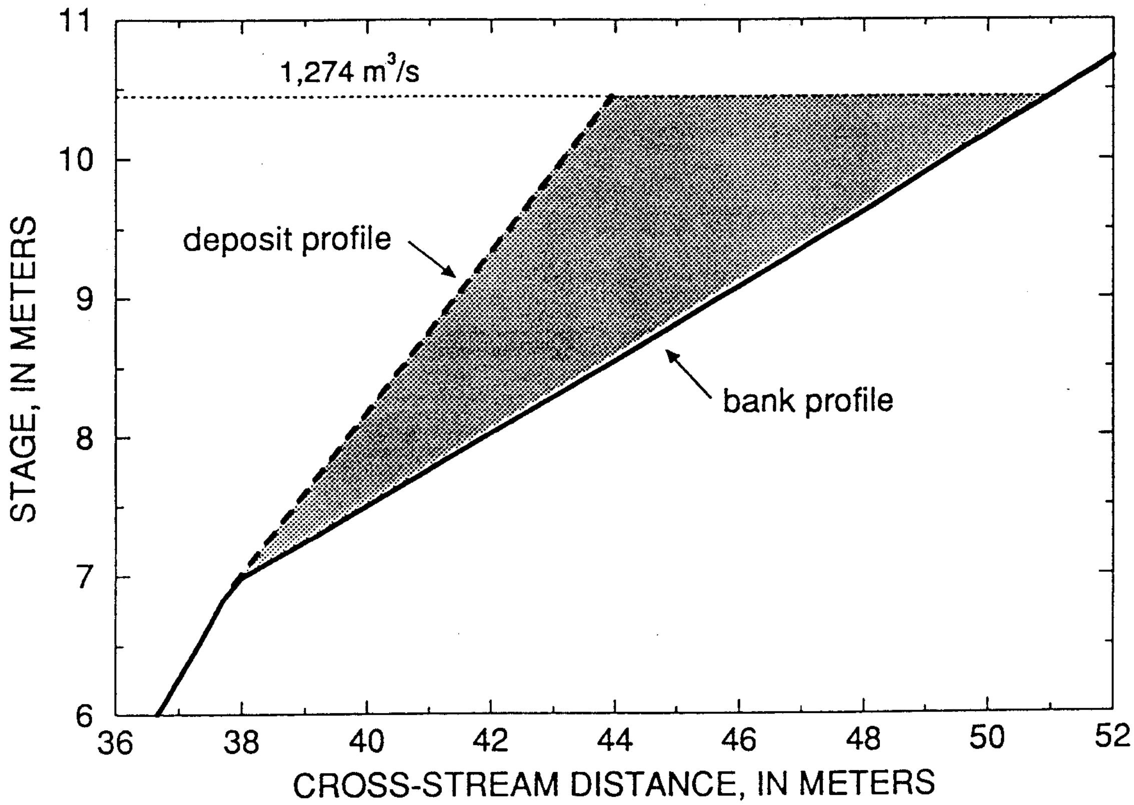
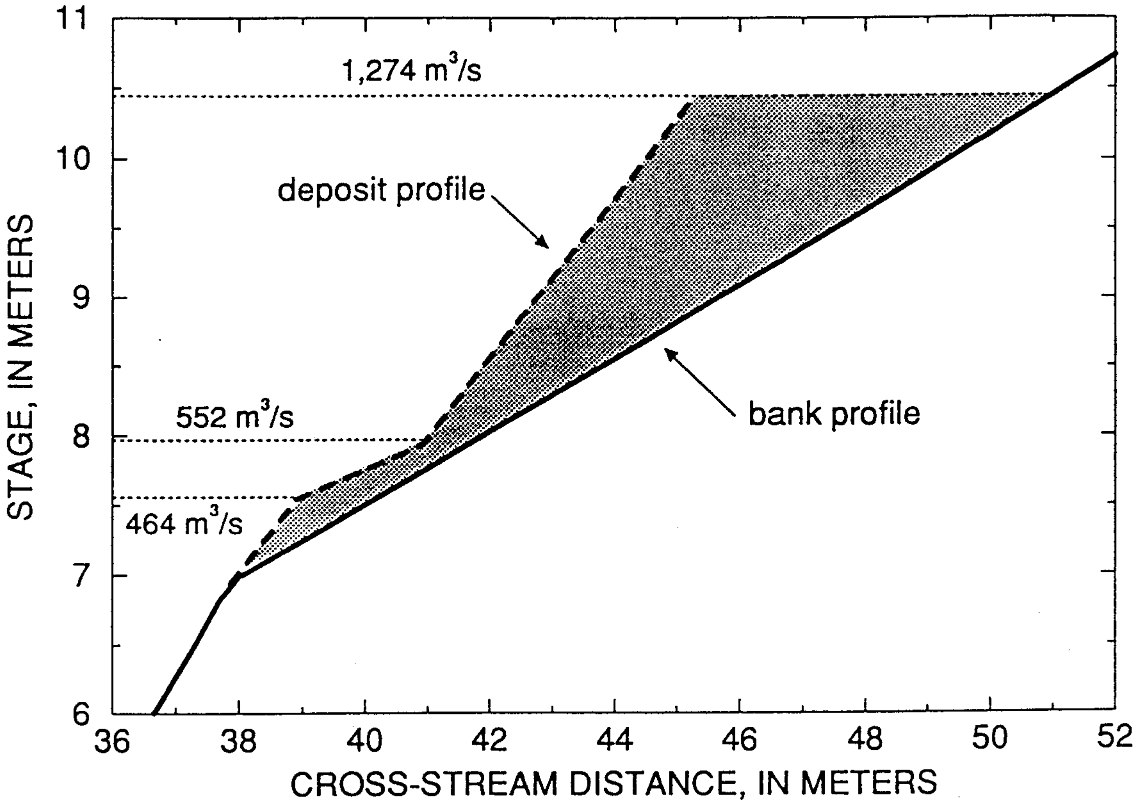

1,274 m³/s
deposit profile
bank profile
STAGE, IN METERS
CROSS-STREAM DISTANCE, IN METERS
11
10
9
8
7
6
36
38
40
42
44
46
48
50
52

Fig 6



1,274 m³/s
deposit profile
552 m³/s
464 m³/s
bank profile
STAGE, IN METERS
CROSS-STREAM DISTANCE, IN METERS
11
10
9
8
7
6
36
38
40
42
44
46
48
50
52

Fig 7

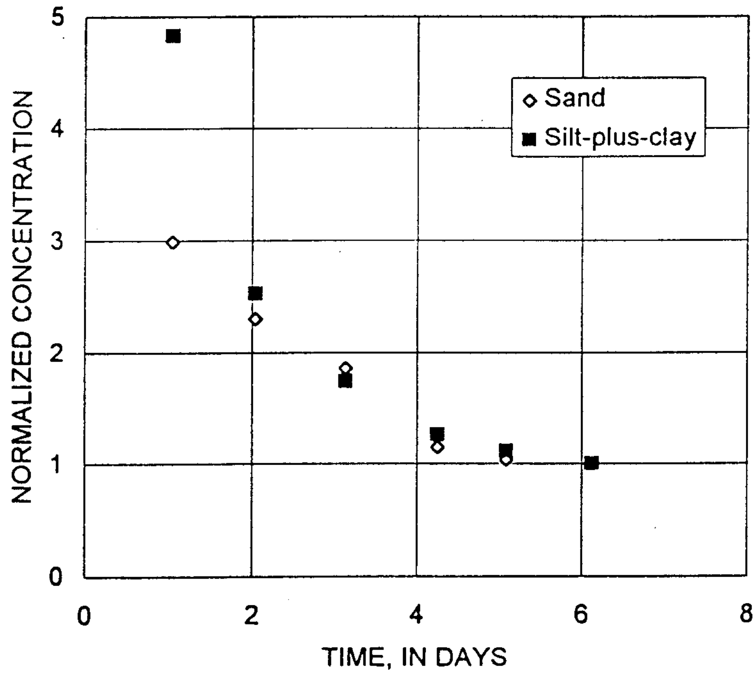

5
4
3
2
1
0
NORMALIZED CONCENTRATION
0
2
4
6
8
TIME, IN DAYS
Sand
Silt-plus-clay

Fig 8

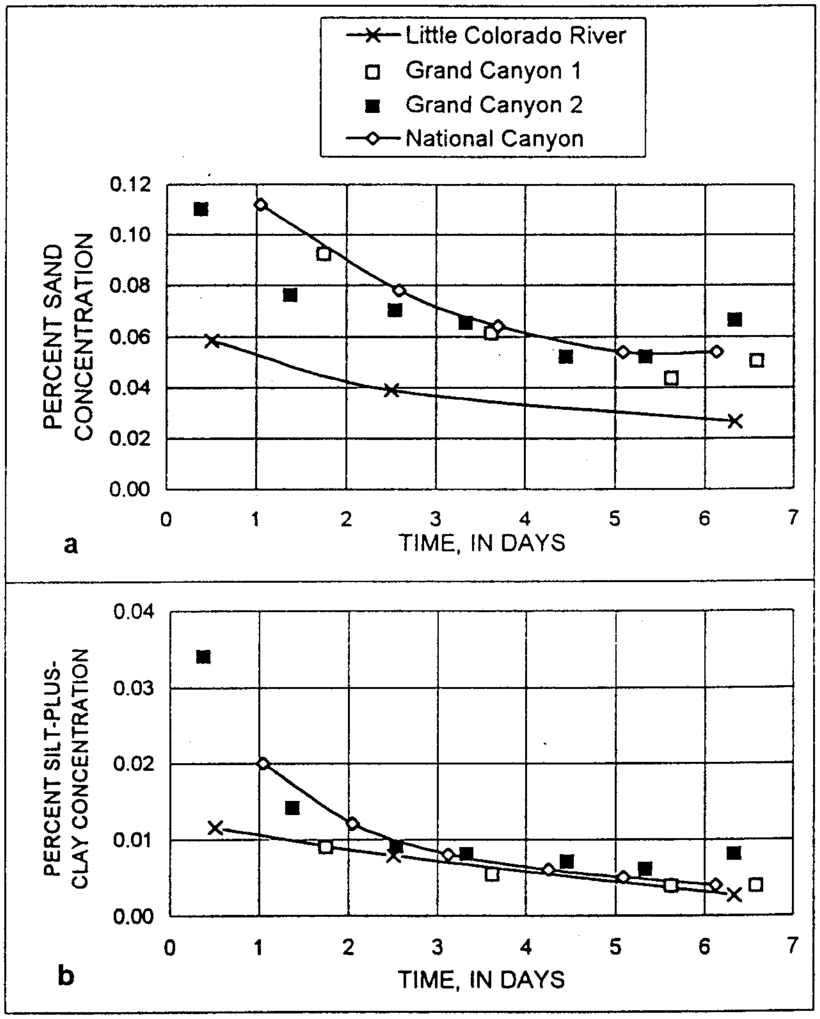

Little Colorado River
Grand Canyon 1
Grand Canyon 2
National Canyon
PERCENT SAND CONCENTRATION
0.12
0.10
0.08
0.06
0.04
0.02
0.00
0 1 2 3 4 5 6 7
TIME, IN DAYS
a
PERCENT SILT-PLUS-CLAY CONCENTRATION
0.04
0.03
0.02
0.01
0.00
0 1 2 3 4 5 6 7
TIME, IN DAYS
b

Fig 9

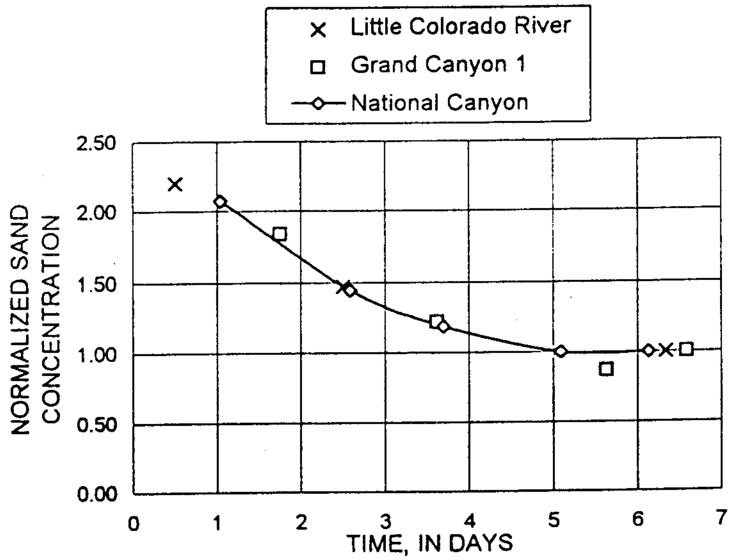

Little Colorado River
Grand Canyon 1
National Canyon
2.50
2.00
1.50
1.00
0.50
0.00
NORMALIZED SAND CONCENTRATION
0
1
2
3
4
5
6
7
TIME, IN DAYS

Fig 10